GEOTECHNICAL ENGINEERING INVESTIGATION
PUERTO SUELLO MULTI-USE PATH PROJECT
MARIN COUNTY, CALIFORNIA

For

NOLTE ASSOCIATES, INC.
1731 N. First Street, Suite A
San Jose, CA 95112

PARIKH CONSULTANTS, INC.
356 S. Milpitas Blvd, Milpitas, CA 95035
(408) 945-1011

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INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed Puerto Suello Multi-Use Path Project in Marin County, California. Our work was performed generally in accordance with the scope of work as per our agreement. The general location of the site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. These recommendations should not be used for bidding purposes or directly for construction cost estimates.

PROPOSED CONSTRUCTION

The proposed project consists of construction of an improved multi-use path that will parallel the existing Route 101 from Mission Avenue to Merrydale Road, as part of the TAM sponsored project. The project will interface with some of the retaining walls and sound walls that are proposed for the Route 101 widening by Caltrans. Based on the design plans provided by Nolte Associates, Inc., the proposed project consists of twelve retaining walls (RW-9 thru RW-20), five sound walls (SW-2, and SW-6 thru SW-9) along the trail alignment. There will be a box culvert extension at approximately Sta. 68+50 and a wing wall extension north of Linden Avenue, along the east side of the existing Union Pacific Railroad (UPRR).

A new tunnel, about 45 m long, is proposed at Lincoln Avenue. Stage construction is anticipated for the proposed tunnel. Groundwater is expected during construction. Temporary/ permanent groundwater control should be provided for the proposed structure. Other features of the project include, but are not limited to, drainage systems with a pumping plant, traffic control systems, signs, utilities relocations, pavement design, landscaping, lighting, etc.

There will also be a new bridge crossing over Linden Avenue. The bridge will be a single-span structure, located in-between the existing UPRR tracks and Route 101. The geotechnical recommendations for the proposed bridge will be submitted in a separate report.

Our recommendations presented in this report are based on the above information. Any major deviations should be reported to our office for further consideration.

PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties, to provide geotechnical recommendations for excavation, retaining walls and sound walls, and to provide foundation design for the project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site, obtaining representative soil samples and logging soil materials encountered in eight exploratory borings, laboratory testing of the collected samples, engineering analysis of the field and laboratory data, and preparation of this report. The basis for this investigation is a set of preliminary plans provided by Nolte Associates, Inc. Caltrans as-built data are available at some of the locations. No new borings were proposed within these segments. The as-built logs of test borings were also attached in Appendix A.

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional geotechnical and engineering services. Therefore, we recommend that a contingency fund be provided to accommodate any technical services during construction that may result from field variations.

SITE CONDITIONS

The project site is parallel to Route 101 from Mission Avenue to Merrydale Avenue. Based on the profile plans provided by the client, the ground elevations gradually increase toward the north. The elevation is ranging from Elev. 6 m at the south end to Elev. 58 m at the north end of the project. UPRR tracks are located along the west side of the proposed trail. The area is mainly occupied by residential properties.

FIELD EXPLORATION

Based on the plans, discussions with design engineer, and readily available data of the area, eight new exploratory borings were drilled to a maximum depth of 14 m (46 feet) below the existing ground surface. The locations and description of the materials are shown on the Site Plan, Plate 2 and Appendix A of the report.

The test borings were advanced with a truck-mounted drill rig using rotary wash drilling method and a portable minuteman rig with solid stem augers. Selected samples were obtained from 64 mm (2.5-inch I.D., Modified California), 35 mm (1.4-inch I.D., Standard Penetration) samplers, and HQ Diamond Core Barrels (64 mm I.D.) at various depths. The MC & SPT samplers were driven into subsurface soils under the impact of a 63.5 kg (140-pound) hammer having a free fall of 76 cm (30 inches). The HQ core barrels were pushed down under hydraulic pressure. The blow counts are presented on the "Log of Test Borings". (When correlating standard penetration data in similar soils, the blow counts for the Modified California Sampler can be taken as roughly twice that for the Standard Penetration Test in similar soils). The samples were sealed and transported to our laboratory for further evaluation and testing. The field investigation was conducted under the supervision of our field engineer who logged the test borings and prepared the samples for subsequent laboratory testing and evaluation.

Laboratory tests were performed on selected samples during field exploration to determine the physical and engineering properties of the subsoils. Laboratory data of moisture contents, total density and unconfined compression tests are presented on the Log of Test Borings.

The bore logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs. The abrupt stratum changes shown on these logs may be gradual and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations.

GEOLOGY

General geologic features pertaining to the site were evaluated by reference to the Geologic Map & Map Database of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties California, by M. C. Blake Jr., 2000 (USGS, MF-2337, Version 1). Based on the publication, the site is generally underlain by Mélange, a tectonic mixture of variably sheared shale and sandstone. A geologic map of the project vicinity is shown in Plate 3.

SUBSOIL CONDITIONS

Based on the investigation and readily available geotechnical information, the subsoils generally consist of firm to stiff lean clay/silty clay interbedded with loose to dense clayey/silty sand layers along the proposed trail.

Between Sta. 65+50 to approximate Sta. 73+00, nine borings (B-3-4, B-3-2, B-1-2, B-2-1, B-2-2, B-6, B-1-1, B-4-2, and B-4) were drilled in 1984 and two CPTs (CPT-1A and CPT-4A) were performed in 2001 by others. The maximum exploration depth is approximate 10.7 m deep below grade. The subsoils within this area generally consist of firm to stiff silty clay/clayey silt overlying loose silty sand. Bedrock was not encountered in these borings. Groundwater was generally encountered at approximately 3 m below existing grade.

Between Sta. 73+00 to 76+00, three borings (B-3, B-2 and B-9) were drilled to a maximum depth of 15 m (1983 and 1984, by others). The subsoils generally consist of dense sand overlying sandy lean clay. Bedrock was encountered in all three borings below the sandy lean clay layer. Based on the boring data, basalt was encountered at Elev. 5 m in B-3 (Sta. 73+40±), and Siltstone was encountered at Elev. 20 m in B-9 (Sta. 75+60±). Groundwater was measured approximately 2 to 8 meters above the soil/rock interface, and it is also lower at the southern end of this section.

From Sta. 76+00 to Sta. 82+00, eight borings (B-8, B-10, B-3, B-1, by others (1984) and TP-7, B-6, B-4, B-5, by Parikh (2006)) and three CPTs (CPT-6, CPT-7 and CPT-11, by others (2001)) were drilled to a maximum depth of 10 meters within this section. Based on the boring data, the subsoils generally consist of stiff lean clay. Bedrock was not encountered in these borings and CPTs. Groundwater was encountered at Elev. 26.3 m and 29.0 m in Boring B-10 and B-3, respectively. Based on the measured water table, groundwater may be anticipated at 2 to 3 m below the existing grade.

North of Sta. 82+00 to about 85+80, the northern end of the project, eight borings (B-3, B-2, B-1, B-8 by Parikh (2006), and D-1 thru D-4, by others (1958)) and four CPTs (CPT-12, CPT-13, CPT-13A and CPT-14A, by others (2001)) were explored within this section. The subsoils generally consist of stiff clay overlying bedrock. Perched water was encountered at the soil/rock interface. Bedrock was encountered at shallow depth (less then 1 meter deep) in B-8 and D-1 thru D-4.

The project also consists of a new sound wall (Sound Wall No. 9) along the east side of HWY 101 between Sta. 66+44± to 83+74±, approximately 1730 m long. No new boring was drilled, and the subsoil conditions were studied based on the thirteen as-built borings (B-9-4 (1983), and B-3-4, B-1-4, B-2-4, B-4-4, B-6-4, B-5-4, B-10-4, B-11-4, B-7-4, B-8-4 (1984), and B-1 (Br. No. 27-50, 1965), and B-1 (Br. No. 27-34, 1983)) provided by the designer. The subsoils generally consist of stiff silty clay/sandy silt with occasional sand lenses south of Linden Avenue. North of Linden Avenue to approximately Sta. 81+00, the borings encountered mostly granular materials and bedrock at various depths. North of the previous section, the boring (B-9-4, 1983 by others) encountered silt clay/clay. Isolated submerged loose sand lenses were encountered, which may be subject to liquefaction during earthquakes. Based on the boring data, groundwater is approximately 3 to 4.5 m (10 to 15 feet) below the existing grade.

It should be noted that the groundwater level at the site may change with passage of time due to groundwater fluctuations from season to season, weather conditions, and other factors which may not have been present at the time of the investigation.

It should be noted that the subsurface conditions described above depict conditions only at the locations indicated on the Site Plan and on the particular date of our investigation. Subsurface conditions at other locations may differ from conditions occurring at the locations explored.

Also, the passage of time may result in a change in the soil conditions at these locations due to environmental and other changes.

EARTHQUAKE CONSIDERATIONS

Seismic Sources

The project site is located in a seismically active part of northern California. Many faults exist in the San Francisco Bay Area that are capable of producing earthquakes that may cause strong ground shaking at the site. Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (1996) are summarized below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

Fault	Distance from Site (km)	Maximum Credible Earthquake Magnitude
San Andreas	~15	8
Hayward	~12	7½

Active faults in the vicinity include the San Andreas Fault and Hayward Fault. A major earthquake on these faults can produce strong ground shaking at the site. A Fault Map of the general project vicinity is shown on Plate 4. Based on the seismic hazard map prepared by Mualchin (1996) and attenuation relationship proposed by Sadigh, et al (1997), a Peak Bedrock Acceleration (PBA) of 0.4 g is anticipated at the site.

Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the proposed bridge structures, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered moderate to high.

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged, cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Clays are generally not susceptible to liquefaction.

Based on the as-built boring data, the groundwater level is relatively high within the project vicinity. Some submerged sporadic loose sand layers/lenses encountered in the borings located at the southern end of the project (B-2-1, B-3-2, B-1-2, B-6, between Sta. 67+00 and 70+80 by others (1984)) may be subject to liquefaction. Surface manifestation of ground failure might not be anticipated. The extent and consequences of this liquefaction, in our opinion, could be limited to some post-liquefaction settlements of the ground surface. It is estimated that the post-liquefaction settlements may be on the order of 25 mm, and probably would be random and localized. For the proposed retaining walls and sound walls within this section, it is recommended to design more joints in between segments.

FINDINGS AND RECOMMENDATIONS

General

Based on the findings of our investigation, it is our opinion that the site is feasible for the planned improvement provided the recommendations presented in this report are incorporated into the final design and construction.

This report was prepared specifically for the proposed structures and roadway according to the plans provided to us. Normal construction procedures were assumed throughout our analysis and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials encountered on the site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

Foundations for Retaining Walls

Per discussion with the designer, the project will require the construction of 11 retaining structures. Information about wall locations, type of walls, design loading case, foundation type, wall length and maximum height are summarized below.

Retaining Wall Summary Table

Wall No.	Wall Type	Design Loading Case	Foundation Type	Approx. Station ("T" Line)	Max. Wall Height (m)	Total Length (m)
9	Type 1	Case I	Spread footing	65+58 to 67+08	1.8	164.3
	Specially designed	--	Spread footing	65+51 to 65+58 67+08 to 67+15	2.1	
10	Type 1	Case II	Spread footing	85+05 to 85+80	3.0	75.0
11	Type 1	Case I	Spread footing	73+13 to 75+76	5.5	203.3
12	Type 1SW (mod.)	Alt. A	CIDH	73+68 to 73+76	2.7 (SW) 7.0 (RW)	19.4
	Type 1	Case I	Spread footing	73+76 to 73+87	3.7 (SW) 1.8 (RW)	
13	Type 1	Case I	Spread footing	81+39 to 81+91	4.8	51.4
14	Type 1	Case I	Spread footing	81+55 to 82+26	5.5	70.7
15	Type 1	Case III	Spread footing	82+00 to 82+20	2.4	19.6
16	Type 1	Case I	Spread footing	82+70 to 83+42	5.5	71.7
17	Type 1	Case I	Spread footing	83+00 to 84+72	5.5	172.5
18	Type 1	Case I	Spread footing	82+27 to 82+33	4.2	6.3
19	Type 1	Case I	Spread footing	82+67 to 82+72	4.2	5.3
20	Mod. Type 1	--	Spread footing	73+95 to 76+00	4.8	205.1

- **Retaining Wall No. 9 – "T" Line, Sta. 65+51± to 67+15± (Bridge No. 27-E0012)**

  Sta. 65+58 and 67+08 Retaining Wall No. 9 is located along the east side of the trail. The proposed trail will be in cut up to 1 m within this section, and the wall is to support the cut slope. It is approximately 164.3 m long with a maximum wall height of 2.1 m.

  Based on Boring B-2-1 (Sound Wall No. 1 & 2, 1984), drilled in the vicinity of the proposed wall, the foundation subgrade soils consist of firm to stiff clay overlying submerged loose sand. Between Sta. 65+58 and 67+08, it is planned to use standard Caltrans Type 1 retaining wall with maximum wall height of 1.8 m (loading case I). In our opinion, the standard Caltrans Type 1 wall supported on spread footing is feasible. The recommended allowable bearing capacity of the subgrade is 110 kPa (2300 psf).

The subsoil condition was interpolated from B-2-1 and B-3-4 (Sound Wall No. 1 & 2, 1984). Based on the boring data, we have assumed that the groundwater is approximately 3 m below the existing grade. Groundwater level can fluctuate may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade, and the pad will also serve as a "load distribution bridge" for reducing loads, differential settlements. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

Sta. 65+51 to 65+58, 67+08 to 67+15 In order to avoid the existing sound wall footings, the proposed wall within these sections is planned to be supported on CIDH concrete piles. According to the load provided by the designer, the pile length will be governed by the lateral demand. The subsoil condition was also interpolated from B-2-1 and B-3-4 (Sound Wall No. 1 & 2, 1984). The saturated loose sand layer encountered in Boring B-2-1 may be subject to liquefaction. Based on the readily available information, the thickness of liquefiable sand along the project limits is generally ranging from 1.5 to 3 m (5 to 10 feet) if encountered. Therefore, we have assumed a soil profile with 1.5 m (5 feet) of clay overlying 3 m (10 feet) of liquefiable sand. Based on the L-Pile results, the designer decided to use 600 mm CIDH piles with pile length of 6.1 m. Plots of deflection, moment, shear and soil reaction along the pile length are attached in Appendix C of the report. For the vertical pile capacity, we have assumed 3 m (10 feet) of liquefiable sand. Down drag force from the upper clay layer was considered, and the capacity within the liquefiable sand layer was neglected.

This wall was originally planned on spread footings. In light of the proposed vertical pile design, it is recommended that further exploration be conducted to verify the design parameters prior to construction. Based on the above, the following table summarizes the pile foundation recommendations.

PILE DATA TABLE

Location	Pile Type	Design Loading (Service)	Nominal Resistance		Cut-off Elev. (m)	Design Tip Elev. (m)	Specified Tip Elev. (m)
			Compression	Tension			
RW#9	600mm CIDH	25 kN	50 kN	--	4.79	-1.3 (1); -1.3 (2)	-1.3
					7.05	0.95 (1); 0.95 (2)	0.95

*Design Tip Elevation is controlled by the following demands: (1) Compression; (2) Lateral Loads

- **Retaining Wall No. 10 - "T" Line, Sta. 85+05± to 85+80±**

The proposed profile will be in cut up to 1 m within this section, and the wall is to support the cut slope along the west side of Los Ranchitos Road. The wall is approximate 75.0 m long with a maximum wall height of 3.0 m.

Based on Boring B-8 (Sound Wall No. 2, 1984) and D-1 thru D-4 (Retaining Wall No. 4 and Sound Wall No. 4, 1958), drilled in the vicinity of the proposed wall, the footings are anticipated to be supported on rock. Bearing capacity is expected to be high within this segment and should exceed the standard design requirement. In our opinion, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for the proposed wall. Perched water was encountered at the soil/rock interface. Water should be expected during the construction.

- **Retaining Wall No. 11 - "T" Line, Sta. 73+73± to 75+76±**

A new breakaway path is proposed at the north side of Linden Lane. The ramp will be sloping down toward south and conform to the existing grade of Linden Lane. Based on the Retaining Wall Plan set provided by the designer, the proposed cut is anticipated up to 4 m. The west side of the breakaway path will be retained by Retaining Wall No. 11, and the east side will be retained by Retaining No. 20 and Sound Wall No. 8.

Based on Boring B-2 (Linden UC, 1983), B-4 and B-9 (Sound Wall No. 2, 1984), drilled in the vicinity of the proposed wall, the foundation subgrade soils consist of dense silty sand. Groundwater was encountered at Elev. 22.4 m in B-9 (Sta. 75+57), and Elev. 11.7 m in B-4

(Sta. 72+56). According to the head difference, groundwater is expected flowing toward south within this section. However, the groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuation, surface and subsurface flows, ground surface run-off, water level in the creek and other factors that may not be present at the time of the investigation.

Based on the boring data, the recommended allowable bearing capacity of the subgrade is approximately 240 kPa (5000 psf). In our opinion, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for Retaining Wall No. 11.

Settlement should not be an issue for the design due to the granular subsoil condition. Based on the boring data, groundwater may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric conforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

– **Retaining Wall No. 12 – "T" Line, Sta. 73+68± to 73+87± (Bridge No. 27-E0013)**

New access stairs are planned to provide access to the trail for the pedestrians from Linden Lane. The stair will be located between the existing UPRR tracks and the proposed trail, located north of Linden Lane. According to the sections provided by the designer, the east side of the existing UPRR embankment will be in cut to accommodate the proposed stairs, and the cut will be supported by Retaining Wall No. 12. The future retaining wall will also retain the new fill on the north side of the stair.

Based on Boring B-2 (Linden UC, 1983), drilled in the vicinity of the proposed wall, the foundation subgrade soils consist of dense sand overlying stiff clay. Groundwater was encountered at Elev. 22.4 m in B-9 (Sta. 75+57), and Elev. 11.7 m in B-4 (Sta. 72+56) in the vicinity. According to the head difference, groundwater is expected to travel toward south and be shallow within this section.

Sta. *73+68 – 73+76* According to the preliminary plans provided by the designer, CIDH concrete piles are considered for the proposed retaining wall in this section due to proximity to UPRR track. According to the boring data, the subgrades below footing consist of approximately 1.5 m of dense sand and 6.1 m of stiff clay overlying bedrock. Groundwater is expected for pile construction. Per discussions with the designer, 600 mm (24-inch) diameter CIDH concrete piles are being considered. Based on our analyses, CIDH concrete piles are appropriate for the foundation of the structure. For 600 mm (24-inch) diameter concrete piles, only shaft friction is considered for pile capacity. The minimum pile spacing at the abutments should be three times the pile diameter. Based on the above, the following table summarizes the pile foundation recommendations.

PILE DATA TABLE

Location	Pile Type	Design Loading (Service)	Nominal Resistance		Cut-off Elev. (m)	Design Tip Elev. (m)	Specified Tip Elev. (m)
			Compression	Tension			
RW#12	600mm CIDH	450 kN	900 kN	--	14	4.8 (1); 6.4 (2)	4.8

*Design Tip Elevation is controlled by the following demands: (1) Compression; (2) Lateral Loads

Per discussion with the designer, lateral load analyses were performed for the planned 600 mm (24-inch) diameter CIDH piles under fixed-head condition. The analyses of the piles should consider group efficiency. A factor of 0.6 (60% of the original soil p-y relationship) is recommended for pile spacing of 3D. Plots of deflection, moment, shear and soil reaction along the pile length are attached in Appendix C of the report.

Sta. *73+76 – 73+87* In our opinion, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for the proposed wall in this section, and the recommended allowable bearing capacity of the subgrade is approximately 240 kPa (5000 psf).

Settlement should not be an issue for the design due to the granular subsoil condition. Based on the boring data, groundwater may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric conforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

- **Retaining Wall No. 12B – "T" Line, Sta. 73+63± to 73+73±**

Retaining Wall No. 12B is located on the east side of the new access stairs, located between the existing UPRR tracks and the proposed trail. The proposed wall is approximately 10.2 m long up to 5.5 m high. The subsoil condition is similar to what was described for Retaining Wall No. 12 with dense sand overlying stiff clay, and groundwater is expected to be shallow within this section. In our opinion, the standard Caltrans Type 1 wall supported on spread footing is feasible. The recommended allowable bearing capacity of the subgrade is 215 kPa (4500 psf).

Settlement should not be an issue for the proposed Retaining Wall No. 12B. Based on the boring data, groundwater may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric conforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

- **Retaining Wall No. 13 – "T" Line, Sta. 81+39± to 81+91±**

Retaining Wall No. 13 is to support the proposed roadway fill up to 4 m that will be placed along the trail. The anticipated length is approximately 51.4 m with a maximum height of about 4.8 m. The wall will be connected to the north end of Sound Wall No. 2. at approx. Sta. 81+39.

Based on Boring B-4 and B-5 (Parikh, 2006), drilled in the vicinity of the proposed wall, the foundation subgrade soils consist of stiff to very stiff clay. In our opinion, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for the proposed wall. The recommended allowable bearing capacity of the subgrade is 165 kPa (3500 psf), which is satisfactory for the proposed wall height up to 4.2 m.

Based on the boring data, groundwater was encountered approximately 6 m (20 feet) below the existing grade. Groundwater can fluctuate and may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade, and the pad will also serve as a "load distribution bridge" for reducing loads, differential settlements. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B). With the 0.6 m thick AB pad, the design allowable bearing capacity could be increased to 190 kPa (4000 psf). Therefore, spread footing is also feasible for the proposed retaining wall for the maximum wall height of 4.8 m.

Based on our evaluation, the maximum anticipated settlements were estimated on the order of less than 13 mm (0.5 inch). Majority of this settlement is in the elastic-overconsolidated range and is anticipated to occur during earthwork construction.

- **Retaining Wall No. 14 - "T" Line, Sta. 81+55± to 82+26±**

Retaining Walls No. 14 is located at the south entrance of proposed tunnel at Lincoln Avenue. The wall is approximately 70.7 m long with maximum wall height of 5.5 m.

Based on Boring B-3 and B-4 (Parikh, 2006), drilled in the vicinity of the proposed wall, the foundation subgrade soils generally consist of firm to stiff clay. In our opinion, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for the proposed wall. For Retaining Wall No. 14, the recommended allowable bearing capacity 165 kPa (3500 psf) for footing below Elev. 41.5 m. This is satisfactory for wall height up to 4.2 m. The recommended allowable bearing capacity is 110 kPa (2300 psf) for footing above Elev. 41.5 m, and this is satisfactory for wall height up to 2.4 m.

Due to the variable subsoil conditions and footing elevations along the wall, it is recommended that more joints be designed in-between segments for the proposed wall. Based on the boring data, groundwater was encountered at Elev. 42 m, approximately 2 m below the existing grade. Groundwater can fluctuate and may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade, and the pad will also serve as a "load distribution bridge" for reducing loads, differential settlements. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

With the 0.6 m thick AB pad, the allowable bearing capacity can be increased to 190 kPa (4000 psf) and 140 kPa (2940 psf) for footing below and above Elev. 41.5 m, respectively. Therefore, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for Retaining Wall No. 14.

Minor fill up to 1.25 m is expected at the southern end of Retaining Wall No. 14. Based on the subsoil condition, settlement should be minimal and is anticipated to occur during earthwork construction. The rest of the trail sections will be in cut.

– **Retaining Wall No. 15 – "T" Line, Sta. 82+00± to 82+20±**

A new ramp is proposed on the west side of the trail, connecting to Lincoln Avenue. Retaining Wall No. 15 will support the cut slope on north side of the ramp. The wall is approximate 19.6 m long with a maximum wall height of 2.4 m.

Based on Boring B-3 (Parikh, 2006), drilled in the vicinity of the proposed wall, the foundation subgrade soils generally consist of stiff clay. The recommended allowable bearing capacity of the subgrade is 165 kPa (3500 psf). In our opinion, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for the proposed wall. This section will be mostly in cut. Therefore, settlement should not be a design issue.

Based on the boring data, groundwater was encountered at Elev. 42 m, approximately 2 m below the existing grade. Groundwater can fluctuate and may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

– **Retaining Wall No. 16 – "T" Line, Sta. 82+71± to 83+42±**

Retaining Walls No. 16 is located on the north entrance of proposed tunnel at Lincoln Avenue. The wall is approximately 71.7 m long with maximum wall height of 5.5 m.

The subsoil condition was interpolated based on Boring B-1 and B-2 (Parikh, 2006), drilled in the vicinity of the proposed wall. The foundation subgrade soils generally consist of firm clay overlying 4 m of stiff clay, underlain by bedrock. Between Sta. 82+71 to 82+87, the recommended allowable bearing capacity is 240 kPa (5000 psf), which is satisfactory for the proposed wall height of 4.8 and 5.5 m. Between Sta. 83+00 to 83+31, the recommended allowable bearing capacity is 105 kPa (2200 psf), which is satisfactory for the proposed wall height of 1.2 m to 2.4 m. Between Sta. 82+87 to 83+00 and 83+31 to 83+49, the recommended bearing capacity is approximately 145 kPa (3000 psf), which is satisfactory for the proposed wall height of 3.0 to 4.2 m. In our opinion, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for the proposed walls.

Based on the boring data, groundwater was encountered at approximately 1 m below the existing grade in both Boring B-1 and B-2 during drilling. Therefore, groundwater can fluctuate and may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade, and the pad will also serve as a "load distribution bridge" for reducing loads, differential settlements. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

This section will be mostly in cut. Settlement should not be an issue for the design. Due to the variable subsoil conditions and footing elevations along the wall, it is recommended that more joints be designed in-between segments for Retaining Wall No. 16.

– **Retaining Wall No. 17 – "T" Line, Sta. 82+00± to 82+20±**

Retaining Wall No. 17 locates along the west side of the trail of approximately 172.5 m long with a maximum wall height of 5.5 m. The trail will be lower than existing grade up to 4 m within this section, and the wall is to support the cut slope.

Based on Boring B-1 and B-8 (Parikh, 2006), drilled in the vicinity of the proposed wall, the foundation subgrade soils consist of stiff clay at the southern end. Boring B-8, located at the northern end of the wall, encountered bedrock at the proposed footing elevation. In our opinion, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for the proposed wall. The recommended allowable bearing capacity of the subgrade is 190 kPa (4000 psf). This section will be mostly in cut. Therefore, settlement should not be an issue for the design.

Based on the boring data, groundwater was encountered approximately 1 m below the existing grade in both borings. Therefore, groundwater may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

– **Retaining Wall No. 18 - "T" Line, Sta. 82+27± to 82+33±**

Retaining Walls No. 18 is located at the south entrance of proposed tunnel at Lincoln Avenue. The proposed wall is approximately 6.3 m long with maximum wall height of 5.5 m.

Based on Boring B-3 (Parikh, 2006), drilled in the vicinity of the proposed wall, the foundation subgrade soils generally consist of firm to stiff clay. Based on the boring data, the recommended bearing capacity is 190 kPa (4000 psf). In our opinion, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for the proposed wall.

Groundwater was encountered at Elev. 42 m, approximately 2 m below the existing grade. Groundwater can fluctuate and may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade, and the pad will also serve as a "load distribution bridge" for reducing loads, differential settlements. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

- **Retaining Wall No. 19 – "T" Line, Sta. 82+67± to 82+72±**

  Retaining Walls No. 19 is located on the north entrance of proposed tunnel at Lincoln Avenue. The wall is approximately 5.3 m long with maximum wall height of 4.2 m.

  Based on Boring B-1 and B-2 (Parikh, 2006), drilled in the vicinity of the proposed wall, the foundation subgrade soils generally consist of firm clay overlying 4 m of stiff clay, underlain by bedrock. The recommended allowable bearing capacity of the subgrade is 165 kPa (3500 psf). In our opinion, standard Caltrans Type 1 retaining wall supported on spread footing is feasible for the proposed walls. This section will be mostly in cut. Settlement should not be an issue for the design.

  Based on the boring data, groundwater was encountered at approximately 1 m below the existing grade in both Boring B-1 and B-2 during drilling. Therefore, groundwater can fluctuate and may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

- **Retaining Wall No. 20 – "T" Line, Sta. 73+95± to 76+00±**

  The proposed Retaining Wall No. 20 will be located north of Linden Avenue, parallel to Retaining Wall No. 11 along the east side of the proposed breakaway path. The wall will be connected to the northern end of Sound Wall No. 8 at Sta. 73+95±. The breakaway path will be sloping down toward south and conform to the existing grade of Linden Lane. The proposed cut is anticipated up to 4 m, and it will be supported by Retaining Walls No. 11 on the west side and Retaining Wall No. 20 Sound Wall No. 8 on the west side.

  Based on Boring B-2 (Linden UC, 1983), B-4 and B-9 (Sound Wall No. 2, 1984), drilled in the vicinity of the proposed wall, the foundation subgrade soils consist of dense silty sand.

Groundwater was encountered at Elev. 22.4 m in B-9 (Sta. 75+57), and Elev. 11.7 m in B-4 (Sta. 72+56). According to the head difference, groundwater is expected flowing toward south within this section. However, the groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuation, surface and subsurface flows, ground surface run-off and other factors that may not be present at the time of the investigation.

According to the Typical Section provided by the designer, the proposed wall will have similar configurations as standard Caltrans Type 1 retaining wall but with Type 736S concrete barriers on top. Therefore, the wall will be a non-standard wall which have to be special-designed. Based on the boring data, it is our opinion that spread footing is feasible for the proposed wall. The recommended allowable bearing capacity of the subgrade is approximately 240 kPa (5000 psf).

Settlement should not be an issue for the design due to the granular subsoil condition. Based on the boring data, groundwater can fluctuate and may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

Foundations for Sound Walls

Per discussion with the designer, our scope of work consists of 5 sound walls. Information about wall locations, type of walls, design loading case, foundation type, wall length and maximum height are summarized below.

Sound Wall Summary Table

<table>
<tr><th>Wall No.</th><th>Wall Type</th><th>Design Loading Case</th><th>Foundation Type</th><th>Approx. Station (along "T" Line)</th><th>Max. Wall Height (m)</th><th>Total Length (m)</th></tr>
<tr><td>2</td><td>Type 1SWB (SW on RW)</td><td>Alt "A"</td><td>CIDH/ Spread footing</td><td>70+97 to 81+61</td><td>4.3 (SW) ~5.5 (RW)</td><td>1004.1</td></tr>
<tr><td>6</td><td>Masonry Block</td><td>Case 1</td><td>CIDH piles</td><td>67+16 to 67+50</td><td>3.7</td><td>34.1</td></tr>
<tr><td>7</td><td>Masonry Block</td><td>Case 1</td><td>CIDH piles</td><td>69+53 to 70+23</td><td>3.7</td><td>69.9</td></tr>
<tr><td rowspan="2">8</td><td>Sound Absorptive on Concrete Barrier</td><td>Case 1</td><td>CIDH piles</td><td>73+27 to 73+48</td><td>3.7</td><td rowspan="2">68.4</td></tr>
<tr><td>Sound Absorptive on Retaining Wall (Mod. Type 1SWB)</td><td>Alt "A"</td><td>Spread Footing</td><td>73+48 to 73+95</td><td>3.7 (SW) 5.5 (RW)</td></tr>
<tr><td>9</td><td>Sound Absorptive on Concrete Barrier</td><td>Case 2</td><td>CIDH piles</td><td>66+44 to 83+74</td><td>3.7</td><td>1729.9</td></tr>
</table>

- **Sound Wall No. 2 – "T" Line, Sta. 70+97± to 81+61±**

For the proposed Sound Wall No. 2, it is planned to use standard Caltrans Retaining Wall Type 1SWB. The wall is located along the west side of the proposed trail. The maximum wall height is expected to be approximately 4.3 m and 5.5 m for the sound wall portion and the retaining wall portion, respectively. Between Sta. 70+97 to 73+08, the wall will be designed by Caltrans, which will not be included in this report. The recommendations for the rest of the wall, north of Sta. 73+08, are presented in the following paragraphs. Based on the subsoil condition along the alignment, spread footing is feasible for the proposed structure.

Sta. 73+08 – 73+40 Based on Boring B-4 (Sound Wall No. 2, 1984), drilled in the vicinity of the section, the foundation subgrade soils consist of stiff silt/clay. The recommended ultimate bearing capacity is approximate 500 kPa (10500 psf). In our opinion, standard Caltrans Retaining Wall Type 1SWB supported on spread footing is feasible for the proposed wall.

Minor fill on the order of 1 m is expected within this section. Based on the subsoil condition, majority of this settlement is in the elastic-overconsolidated range and is anticipated to occur during earthwork construction. Therefore, settlement should not be an issue for the design.

Based on the boring data, we have assumed that the groundwater is approximately 5 m below the existing grade, which is about 2 to 3 m below the bottom of the footings. Groundwater can fluctuate and may be expected. If groundwater were present during construction, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

Sta. 73+80 – 76+00 Based on Boring B-2 (Linden UC, 1983) and B-9 (Sound Wall No. 2, 1984), drilled in the vicinity of this section, the foundation subgrade soils consist of dense sand over stiff clay. The recommended ultimate bearing capacity is approximate 720 kPa (15000 psf). In our opinion, standard Caltrans Retaining Wall Type 1SWB supported on spread footing is feasible for the proposed wall.

Fill up to 2 to 3 m is expected within this section. Based on the boring data, majority of this settlement is in the elastic-overconsolidated range and is anticipated to occur during earthwork construction. Therefore, settlement should not be an issue for the design.

Based on the boring data, the groundwater is ranging from 3 to 5 m below the existing grade. Groundwater may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

Sta. 76+00 – 78+60 Based on Boring B-8, B-10 (Sound Wall No. 2, 1984) and CPT-7 (Sound Wall No. 2, 2001), drilled in the vicinity of the proposed wall, the foundation subgrade soils generally consist of stiff clay. The recommended ultimate bearing capacity is approximate 480 kPa (10000 psf). In our opinion, standard Caltrans Retaining Wall Type 1SWB supported on spread footing is feasible for the proposed wall.

Fill up to 2 to 3 m is expected within this section. Based on the boring data, majority of this settlement is in the elastic-overconsolidated range and is anticipated to occur during earthwork construction. Therefore, settlement should not be an issue for the design.

Based on the boring data, the groundwater is approximately 2 to 5 m below the existing grade. Groundwater may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

Sta. 78+60 – 81+61 Based on Boring B-1, B-3 (Sound Wall No. 2, 1984) and B-5, B-6 (Parikh, 2006), drilled in the vicinity of the proposed wall, the foundation subgrade soils generally consist of stiff clay. The recommended ultimate bearing capacity is approximate 480 kPa (10000 psf). In our opinion, standard Caltrans Retaining Wall Type 1SWB supported on spread footing is feasible for the proposed wall.

Fill up to 2 to 3 m is also expected within this section. Based on the boring data, majority of this settlement is in the elastic-overconsolidated range and is anticipated to occur during earthwork construction. Therefore, settlement should not be an issue for the design.

Based on the boring data, groundwater was encountered at 4 m below the existing grade at the south end in B-3, and 6 m at the north end in B-4. Groundwater might be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric comforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

- **Sound Wall No. 6 – "T" Line, Sta. 67+16± to 67+50±**

The proposed sound wall is approximate 34 m in length with maximum wall height of 3.7 m. It is planned to use standard Caltrans Sound Wall – Masonry Block on Type 736 Barrier (see

Caltrans Standard Plans B15-6 thru B15-8). The wall will be supported on barriers and CIDH concrete piles.

Based on Boring B-2-1 (Sound Wall No. 1 & 2, 1984), drilled in the vicinity of the proposed wall, the foundation subgrade soils consist of approximately 3 m of firm to stiff lean clay overlying submerged loose sand. The submerged loose sand layer may be subject to liquefaction during earthquake, and post-liquefaction settlement may be expected. The estimated settlement is on the order of 25 mm, and probably would be random and localized. Groundwater is approximately 3 m below the existing grade. Groundwater may be expected during foundation construction.

For design of sound wall foundations, an angle of shearing resistance (ϕ) of 30 degrees may be used. The recommended allowable net lateral soil pressure is approximate 100 $kN/m^2/m$ (635 psf/ft). Per typical section provided by the designer, the ground is expected to be level on either side of the wall. Therefore, Case 1 design for CIDH piles is applicable. Based on the assumption above and the Caltrans Standard Plans, the proposed CIDH pile properties can be summarized as follow:

- Design case = Case 1
- Angle of shearing resistance, $\phi = 30$ degrees
- Pile diameter = 400 mm
- Pile Length, L = 2600 mm
- Pile spacing, S = 3000 mm

Per discussion with the reviewer, the on-site soil might consist of expansive soil, which may pose impact on the light-weighted sound wall structure. It is recommended to place 0.3 m (1 foot) thick of structural backfill under the wall bottom per Caltrans Standard Specifications (Section 19-3.06).

– **Sound Wall No. 7 - "T" Line, Sta. 69+53± to 70+23±**

The proposed sound wall is approximate 70 m in length with maximum wall height of 3.7 m. It is planned to use standard Caltrans Sound Wall – Masonry Block on Type 736 Barrier (see

Caltrans Standard Plans B15-6 thru B16-8). The wall will be supported on barrier and CIDH concrete piles.

Based on Boring B-7 (Sound Wall No. 1 & 2, 1984), drilled in the vicinity of the proposed wall, the foundation subgrade soils consist of silt/clay. Groundwater is approximately 3 m below the existing grade. Groundwater may be expected during construction.

For design of sound wall foundations, an angle of shearing resistance (ϕ) of 25 degrees may be used. The recommended allowable net lateral soil pressure is approximate 47.1 $kN/m^2/m$ (300 psf/ft). Per typical section provided by the designer, the ground is expected to be level on either side of the wall. Therefore, Case 1 design for CIDH piles is applicable. Based on the assumption above and the Caltrans Standard Plans, the proposed CIDH pile properties can be summarized as follow:

- Design case = Case 1
- Angle of shearing resistance, $\phi = 25$ degrees
- Pile diameter = 400 mm
- Pile Length, L = 3100 mm
- Pile spacing, S = 3000 mm

Per discussion with the reviewer, the on-site soil might consist of expansive soil, which may pose impact on the light-weighted sound wall structure. It is recommended to place 0.3 m (1 foot) thick of structural backfill under the wall bottom per Caltrans Standard Specifications (Section 19-3.06).

- **Sound Wall No. 8 – "T" Line, Sta. 73+27± to 73+95±**

Sta. 73+27 – 73+48 The proposed sound wall within this section is approximate 21.2 m in length with wall height of 3.7 m. It is planned to use "Port-O-Wall" absorptive sound wall system on Type 736S concrete barriers. Per discussion with the designer, the pile design will be determined based on standard Caltrans Sound Wall – Masonry Block on Type 736 Barrier (see Caltrans Standard Plans B15-6 thru B15-8). The wall is expected to be supported on CIDH concrete piles.

The proposed footing depth is above Boring B-3 (Linden UC, 1983), drilled in the vicinity of the proposed wall. Therefore, it is prudent to assume a lower angle of shearing resistance (ϕ) of 25 degrees for the proposed sound wall foundations. The recommended allowable net lateral soil pressure is approximate 125 $kN/m^2/m$ (800 psf/ft). Per typical section provided by the designer, the ground is expected to be level on either side of the wall. Therefore, Case 1 design for CIDH piles is applicable. Based on the assumption above and the Bridge Design Details Sheets, the proposed CIDH pile properties can be summarized as follow:

- Design case = Case 1
- Angle of shearing resistance, ϕ = 25 degrees
- Pile diameter = 400 mm
- Pile Length, L = 3100 mm
- Pile spacing, S = 3000 mm

Per discussion with the reviewer, the on-site soil might consist of expansive soil, which may pose impact on the light-weighted sound wall structure. It is recommended to place 0.3 m (1 foot) thick of structural backfill under the wall bottom per Caltrans Standard Specifications (Section 19-3.06).

Sta. 73+48 – 73+53 The proposed wall within this section is approximately 5.3 m long with maximum wall height of 4.2 m for the retaining wall portion, located south of Linden Lane. Based on Boring B-3 (Linden UC, 1983), drilled in the vicinity of the proposed wall, the foundation subgrade soils generally consist of stiff clay. It is planned to use the "Port-O-Wall" absorptive sound wall system on Type 736SV concrete barriers on retaining wall. In our opinion, it is reasonable to use the same design criteria as the standard Caltrans Retaining Wall Type 1SWB supported on spread footing. The recommended ultimate bearing capacity is approximate 395 kPa (8250 psf). Spread footings are feasible for the proposed wall.

Based on the boring data, groundwater is anticipated to be shallow. Groundwater can fluctuate and may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric conforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

Sta. 73+69 – 73+95 The proposed sound wall on retaining wall is approximately 26.1 m long with maximum retaining wall height of 5.5 m. Based on Boring B-2 (Linden UC, 1983), drilled in the vicinity of the proposed wall, the foundation subgrade soils generally consist of dense sand overlying stiff clay. "Port-O-Wall" absorptive sound wall system will be used for the proposed wall. In our opinion, it is reasonable to use the same design criteria as the standard Caltrans Retaining Wall Type 1SWB supported on spread footing. The recommended ultimate bearing capacity is approximate 480 kPa (10000 psf). Spread footings are feasible for the proposed wall.

Based on the boring data, groundwater is anticipated to be even shallower north of Linden Avenue. Groundwater can fluctuate and may be expected during footing excavation. Therefore, it is recommended to construct a working platform to stabilize the subgrade. This platform should consist of a minimum of 0.6 m (24 inches) of Aggregate Base rock overlying a layer of high strength geotextile fabric conforming to Caltrans Standard Specifications (Section 88-1.04 Type B).

– **Sound Wall No. 9 – "T" Line, Sta. 66+44± to 83+74±**

Based on the Caltrans as-built plan (dated October, 1987), there is an existing sound wall (Sound Wall No. 4) along the east side of HWY 101 within the project vicinity. Per discussion with the designer, it is planned to remove and replace portions of the existing sound wall. At some location, the existing wall will be left in place, and the new sound wall will be built parallel to it. The proposed sound wall is approximate 1730 m in length with maximum wall height of 3.7 m.

No new boring was drilled for the proposed Sound Wall No. 9. Our recommendations are based on the as-built information. Along the wall alignment, thirteen borings were explored in 1965, 1983 & 1984. Based on the boring data, the subsoils generally consist of stiff silty clay/sandy silt with occasional sand lenses south of Linden Avenue. North of Linden Avenue to approximately Sta. 81+00, the borings encountered mostly granular materials and bedrock at various depths. North of the previous section, the boring (B-9-4, 1983 by others) encountered silt clay/clay. Isolated submerged loose sand lenses were encountered, which may be subject to liquefaction during earthquakes. Post-liquefaction settlement may be expected, and the estimated settlement is on the order of 25 mm, and probably would be random and localized.

Based on the boring data, groundwater is approximately 3 to 4.5 m (10 to 15 feet) below the existing grade. Groundwater can fluctuate and may be expected during foundation construction.

Based on the Sound Wall Plans and the Typical Sections provided by the designer, it is planned to utilize the "Port-O-Wall" aborptive sound wall system on Type 736S concrete barriers. It is reasonable to use the same design criteria as the standard Caltrans Sound Wall – Masonry Block on Type 736 Barrier (see Caltrans Standard Plans B15-6 thru B15-8). Therefore, the proposed wall will be supported on barriers and CIDH piles.

Per typical section provided by the designer, Case 2 (sloping ground) condition is expected. The recommended allowable net lateral soil pressure is approximate 13 $kN/m^2/m$ (80 psf/ft). Based on the assumption above and the Caltrans Standard Plans, an angle of shearing resistance of 30 degrees may be used to determine the CIDH pile properties. Detailed pile dimensions, such as length and spacing, will be determined by the designer based on the layout of the future sound wall.

Per discussion with the reviewer, the on-site soil might consist of expansive soil, which may pose impact on the light-weighted sound wall structure. It is recommended to place 0.3 m (1 foot) thick of structural backfill under the wall bottom per Caltrans Standard Specifications (Section 19-3.06).

Portion of the proposed Sound Wall No. 9 will be built immediately in front of the existing wall. Per California Trenching and Shoring Manual, the distributed load per pile may be increased by an adjustment factor (arching capability) of 0.08 times the internal friction angle of the soil (0.08ϕ), but not to exceed a value of 3.0. Based on the boring data, the recommended Angle of Shearing Resistance for design is 30° for Sound Wall No. 9. The arching capability factor would be 2.4 for the proposed piles (i.e. the effective width will be 2.4 times the pile diameter). The effective width of a single 400 mm (16 inches) diameter CIDH pile will be 1 m (3.2 feet). Currently, the plan is showing a 0.9 m minimum stagger. As per the designer, it is required for the contractor to expose the existing sound wall piles. It is our opinion that the impact should be relatively insignificant and it is acceptable for the proposed sound wall.

Per discussion with the designer and the Sound Wall Plans provided, the new wall and the existing wall will not be structurally connected. In our opinion, most likely the two walls will not resist the same wind load simultaneously. The impact of the extra load due to the adjacent sound wall should be minimal.

Culvert Crossings at Sta. 68+93, 69+92, 70+72 and 72+08 Along Sound Wall No. 9, there are four drainage culvert crossings between Sta. 68+90 and 72+10. Based on the plans provided by the designer, the sound wall at these locations will be modified and bridged over the existing culverts. The existing sound wall at these locations will be removed, and the CIDH concrete piles of the proposed sound wall will be re-arranged.

Per discussion with the designer, Caltrans standard 400 mm (16 inches) diameter CIDH concrete piles are considered. According to the boring data (B-2-4, B-4-4, 1984 and B-1, 1965 by others), the subsurface conditions consist of cohesive material up to 5 m deep. The design allowable vertical load is 115 kN (13 tons), and the lateral load is 58 kN (13 kips). It is our understanding the connection between the pile head and the pile cap will be "fixed-head" condition. Based on our analyses, CIDH concrete piles are feasible for the foundation of the structures. According to the design load provided by the designer, the recommended pile length is 4.6 m (15 feet) long. For the lateral capacity, we have considered group efficiency in our analysis. Plots of deflection, moment, shear and soil reaction along the pile length are attached in Appendix C of the report.

Per discussion with the designer (Nolte Associates, Inc.), the existing sound walls at the culvert crossings will be removed. The existing piles will be left in place and will not be connected to the new foundation system. In our opinion, the new foundation should pose minimal impact to the existing culverts. As per the designer, it is required for the contractor to expose the existing sound wall piles. The new piles will be installed with minimum 0.910 m spacing based on the locations of the existing piles.

Groundwater is expected for pile construction. The use of full-length temporary steel casing should be expected to keep the drilled hole open to assure the integrity of the proposed 400 mm diameter CIDH concrete piles.

Sound Wall on Retaining Wall at Sta. 77+90 to 78+76 Between Sta. 77+90 to 78+76, the finished grad on one side of the sound wall is approximately 2.4 m higher than the other side. Therefore, the Caltrans standard sound wall is not feasible within this section. The new sound wall will be constructed immediately in front of an existing sound wall. Per discussion with the designer, it is planned to use modified Caltrans standard retaining wall type 5SWB (sound wall on retaining wall). CIDH concrete piles will be used for foundation support to minimize the lateral impact on the existing sound wall.

It is planned to use Caltrans standard 400 mm (16 inches) diameter CIDH concrete piles. According to the as-built boring data (B-7-4 and B-11-4, 1984, by others), the subsurface conditions consist of approximate 4.5 m of cohesive material overlying Shale. The design allowable vertical load is 400 kN (45 tons). Based on our analyses, CIDH concrete piles are feasible for the foundation of the structures. According to the design load provided by the designer, the recommended pile length is 7.6 m (25 feet) long. For the lateral capacity, we have considered group efficiency in our analysis. Plots of deflection, moment, shear and soil reaction along the pile length are attached in Appendix C of the report.

Groundwater is expected for pile construction. The use of full-length temporary steel casing should be expected to keep the drilled hole open to assure the integrity of the proposed 400 mm diameter CIDH concrete piles. Based on the as-built boring data, hard drilling condition may be expected during construction.

Cast-In-Drilled-Hole (CIDH) Concrete Pile

Caltrans standard specifications for "Cast-In-Place Concrete Piling" should be followed for all the proposed CIDH piles. If sand and gravel layers were encountered, localized hard drilling and raveling or caving may be expected. This may require additional drilling and cleaning effort and may increase the concrete volume for the piles. Groundwater level may fluctuate and perched water condition may be encountered. It is prudent to make the contractor aware of these conditions so that he takes appropriate steps to comply with the standards and maintain the integrity of the CIDH concrete piles.

In our opinion, the pile excavations may require temporary steel casing or similar measures to maintain the holes open. Access tubes are required to allow for construction quality control (gamma-gamma logging) in all CIDH piles that are 600 mm (24") diameter or larger, except when the holes are dewatered without the use of temporary casing to control groundwater. For the 400 mm CIDH concrete piles, it is not feasible to install access tubes for quality control purpose (gamma-gamma logging). Therefore, if groundwater were encountered, full-length casing should be used during construction to assure the pile integrity. Contractor should be prepared for possible caving due to sand and gravel layers. All piles excavation should be observed by the geotechnical engineer or regulatory agency prior to the placement of the reinforcement and concrete so that if conditions differ from those anticipated, appropriate recommendations can be made.

Tunnel at Lincoln Avenue

The boring data (B-2 and B-3, Parikh 2006) indicated stiff clay was encountered below the tunnel bottom. For the proposed tunnel, box culvert-type-of structure may be feasible for the project. It is planned to use cut-and-cover construction method and stage construction is anticipated. The planned tunnel invert is at Elev. 41.5 m at the south end, and Elev. 43.5 m at the north end. For preliminary information, the recommended allowable bearing capacity is 96 kPa (2000 psf).

Perched water was encountered at Elev. 41.8 m and 45.9 m at B-3 (south end) and B-2 (north end), respectively. Water is expected during tunnel excavation. It is recommended to have a permanent dewatering system, such as deep wells and pump station. Water cut-off wall at the site is also recommended to lower the ground water level below the proposed tunnel invert.

The tunnel can also be designed for groundwater. If so, the tunnel and the wing walls at both ends of the tunnel should be water proof, and tension piles or tiedowns should be utilized to account for the buoyancy.

In order to provide a uniform foundation support for the tunnel, it is recommended that the proposed tunnel subgrade be improved as follows:

1. Over-excavate 0.6 m (2 feet) below the invert. The excavation should extend 0.6 m (2 feet) beyond the culvert footprint.

2. Place a layer of reinforcing geotextile fabric conforming to Caltrans standard specifications (Section 88-1.04) at the subgrade.

3. Place 0.6 m (2 feet) of Class 2 Aggregate Base (AB) over the geotextile fabric. The AB should be compacted to min. 95% relative compaction per ASTM D-1557. The geotextile fabric should then be wrapped with a minimum 0.6 m (2 feet) of fabric overlap on top of the compacted AB.

The layer of compacted AB with geotextile fabric serves as a "load distribution bridge" for reducing loads, differential settlements, and provides a working platform.

The tunnel should be designed for at-rest soil condition plus live loads. The Applied Lateral Earth Pressures is summarized below. The recommended design groundwater level is at 2 m below the crown of the tunnel. Water pressure 9.8 kN/m^3 (62.4 pcf) should be added for below groundwater condition.

Applied Lateral Earth Pressure

- Active Condition — 5.7 kN/m^3 (36 pcf) Equivalent Fluid Pressure (EFP) for engineered fill.
- At-Rest Condition — 8.6 kN/m^3 (55 pcf) EFP for engineered fill above groundwater
4.7 kN/m^3 (30 pcf) EFP for engineered fill below groundwater

Culvert Extension at Sta. 68+50±

For the proposed trail, the existing box culvert at Sta. 68+50± will be extended. According to the drainage detail plans provided by the designer, the bottom of the culvert extension is assumed to be approximately at Elev. 6.5 m. Based on the boring data (B-3-2, B-2-2, and B-4-2, by others (1984)), the subsoils generally consist of medium dense silty sand. For the proposed culvert extension, the recommended allowable bearing capacity is 144 kPa (3000 psf). Submerged medium dense sand lenses were encountered, which might be subjected to liquefaction during earthquakes. Post-liquefaction settlement might be expected on the order of 25 mm (1 inch).

Groundwater was encountered at Elev. 6.6 m at B-3-2. Water is expected during construction. In order to provide a uniform foundation support for the tunnel, it is recommended that the proposed tunnel subgrade be improved as follows:

1. Over-excavate 0.6 m (2 feet) below the invert. The excavation should extend 0.6 m (2 feet) beyond the culvert footprint.

2. Place a layer of reinforcing geotextile fabric conforming to Caltrans standard specifications (Section 88-1.04) at the subgrade.

3. Place 0.6 m (2 feet) of Class 2 Aggregate Base (AB) over the geotextile fabric. The AB should be compacted to min. 95% relative compaction per ASTM D-1557. The geotextile fabric should then be wrapped with a minimum 0.6 m (2 feet) of fabric overlap on top of the compacted AB.

The layer of compacted AB with geotextile fabric serves as a "load distribution bridge" for reducing loads, differential settlements, and provides a working platform.

The proposed culvert should be designed for at-rest soil condition plus live loads. The Applied Lateral Earth Pressures is summarized below. Per boring data, it is prudent to assume the recommended design groundwater level at Elev. 6 m. Water pressure 9.8 kN/m^3 (62.4 pcf) should be added for below groundwater condition.

Applied Lateral Earth Pressure

- Active Condition — 5.7 kN/m^3 (36 pcf) Equivalent Fluid Pressure (EFP) for engineered fill.
- At-Rest Condition — 8.6 kN/m^3 (55 pcf) EFP for engineered fill above groundwater
  4.7 kN/m^3 (30 pcf) EFP for engineered fill below groundwater

Dewatering

Based on the boring data, groundwater was encountered above the invert elevation of the proposed tunnel. Dewatering may be required during construction. Dewatering of excavations is normally the responsibility of the contractor and should be a design-build system. It appears that shallow sumps may be used when needed. The contractor should satisfy themselves with the prevailing conditions and obtain additional information if needed to design the required dewatering method.

Controlled dewatering should be performed to avoid possible piping/blowout at the base of excavation or to avoid excessive settlement problems in the surrounding areas. All dewatering systems should be properly designed to prevent pumping soil fines with the discharge water. This work should be closely monitored during construction. If soil fines are pumped, the contractor should revise his dewatering operations. Otherwise, failure of shoring wall, partial instability of excavation bottom resulting in intolerable settlement of the surrounding ground (affecting existing foundations and utilities), and unsafe working conditions may occur. The contractor should provide sampling locations for discharge of each pump for their monitoring purposes.

Grading

It is anticipated that grading of the project will include excavation up to about 5 m and fill up to 4 m for the proposed trail. All grading operations should be performed in accordance with the project specifications and Caltrans standards. A representative from our office or the regulatory agency should observe the grading operation and perform moisture and density tests on prepared subgrade and compacted fill. Should there be any alterations of the proposed construction that will affect the stated bases of our recommendations, we should be informed so that we can review such changes and amend or submit additional recommendations.

Areas to receive engineered fill should be excavated to remove any and all loose/soft soils. The resulting surface upon which fill is to be placed should be observed by the engineer. Areas receiving fill should be scarified, moisture conditioned and compacted in accordance with the project specifications.

Engineered Fill. Engineered fill should be non-expansive and consist of relatively granular material having a P.I. of less than 20 and a minimum Sand Equivalent of 8. In addition, we recommend that the material within 1 m of the proposed pavement subgrade have a minimum R-value of 15. The on-site soils, if free of organic or other deleterious material, may be used as engineered fill provided they meet the above criteria. The materials directly behind the bridge abutment walls and retaining walls should consist of Structure Backfill conforming to Caltrans standard specifications (Section 19-3.06).

Compaction of Fill and Subgrade. The project specific recommendations for required compaction as per Caltrans standard specifications are as follows:

- 90% for subgrade preparation, general fill and backfilling after removing buried utilities and depressions caused due to construction activities, etc.

- 95% for all engineered fill for structural backfill of bridge abutments and for upper 15 cm (6 inches) of pavement subgrade and aggregate base of pavement sections.

Slope Ratios. For temporary cuts above groundwater level, a slope ratio of up to 1H:1V may be used. It should be noted that during excavation local irregularities such as sand and gravel pockets and layers may be encountered. This may require flattening of the slope or shoring. For permanent fill slope, a slope ratio of 2H:1V is recommended. For temporary fill slope, a slope ratio of 1H:1V may be used.

Our office should review the final grading plans prior to grading to see that the intent of our recommendations is included in the plans.

Structural Pavement

For the proposed trail, three R-value tests were performed on selected samples to determine design values. Based on the laboratory tests, the R-values are 10 and 12 for Fat Clay and 20 for the Lean Clay. Generally, a lower R-value is used for a pavement design. Based on the test results and the subgrade condition, it is reasonable to use R-value of 10 for the pavement design. Per discussion with the designer, Traffic Index (TI) is not available at this moment but is expected to be minimal for the project. For preliminary cost estimation purpose, 75 mm (3") Asphalt Concrete over 200 (8") Aggregate Base should be feasible for the proposed multi-use path.

Corrosion

No corrosion test result is available from the as-built boring data. Two corrosion tests were performed on selected samples obtained from B-2 and B-5, drilled by Parikh consultants, Inc. in February 2006. B-2 and B-5 are located at the intersection of Lincoln Avenue and the proposed trail, approximately 0.9 km north of the project site. The corrosion investigations performed are in general accordance with the provisions of California Test Method 643. A summary of the corrosion test data is presented below.

Boring	Depth (m)	PH	Resistivity (ohms-cm)	Sulfate (ppm)	Chloride (ppm)
B-2	3.4	6.81	2570	61.0	5.7
B-5	1.8	6.94	2600	100.6	8.9

Based on the data, the site is considered non-corrosive per Caltrans corrosion design guideline, and standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructures. The minimum cement factor and cover thickness should be per Caltrans Bridge Design Specifications (Section 8.22).

Plan Review

We recommend that final plans for construction be reviewed by this office prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

Construction Observation

To a degree, the performance of any structure is dependent upon construction procedures and quality. Hence, observation of foundation excavations, subgrade preparation and pile installation should be carried out by the geotechnical engineer or the appropriate regulating agencies. If the subsurface conditions different from those forming the basis of our recommendations is encountered this office should be informed in order to assess the need for design changes. Therefore, the recommendations presented in this report are contingent upon good quality control and these geotechnical observations during construction.

INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our site reconnaissance and the assumption that the subsurface conditions do not deviate from observed conditions. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings. The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

This report has been prepared for the proposed grade separation project as described earlier, to assist the engineer in the design of this project. In the event any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered during construction, our conclusions and recommendations shall not be considered valid unless the changes or variations are reviewed and our recommendations modified or approved by us in writing.

This report is issued with the understanding that it is the designer's responsibility to ensure that the information and recommendations contained herein are incorporated into the project and that necessary steps are also taken to see that the recommendations are carried out in the field.

The findings in this report are valid as of the present date. However, changes in the subsurface conditions can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge. Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Very Truly Yours,
PARIKH CONSULTANTS, INC.

Frank Y. Wang, P.E. 67751
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
GAURANG R. PARIKH
No. 666
EXP. 12-31-07
GEOTECHNICAL
STATE OF CALIFORNIA

Gary Parikh, P.E., G.E 666
Project Manager

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